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# SEISMIC RESPONSE OF NORMALLY CONSOLIDATED COHESIVE SOILS IN GENTLY INCLINED SUBMERGED SLOPES

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## ABSTRACT

The geological profile of submerged slopes on the continental shelf typically includes soft cohesive soils with layer thicknesses ranging from a few meters to tens or hundreds of meters. The response of these soils in simple shear tests is largely influenced by the presence of an initial consolidation shear stress, inducing anisotropic stress-strain-strength properties which depend also on the direction of shear. In this paper, a new simplified effective-stress-based model describing the behavior of normally to lightly overconsolidated cohesive soils is used in conjunction with a one-dimensional seismic site response analysis computer code to illustrate the importance of accounting for anisotropy, small strain nonlinearity and pore pressure development. In particular, a simple example is carried out to compare results for level ground conditions and a 10° slope. Depth profiling of the maximum shear strains and permanent deformations provide insight into the mechanisms of deformation during a seismic event, and the effects of sloping ground conditions.

## INTRODUCTION

Seismic site response analyses of submerged slopes in the continental shelf have become an important element in the risk assessment and prediction of performance for offshore structures worldwide. The typical geological profile is characterized by parallel layers of normally consolidated to lightly overconsolidated clay deposits. Submarine slope failures attributed to seismic loading can reach large sizes, up to kilometers both in width and length, and have been reported to occur on slopes inclined 5 degrees or less to the horizontal (e.g., Frydman et al. 1988).

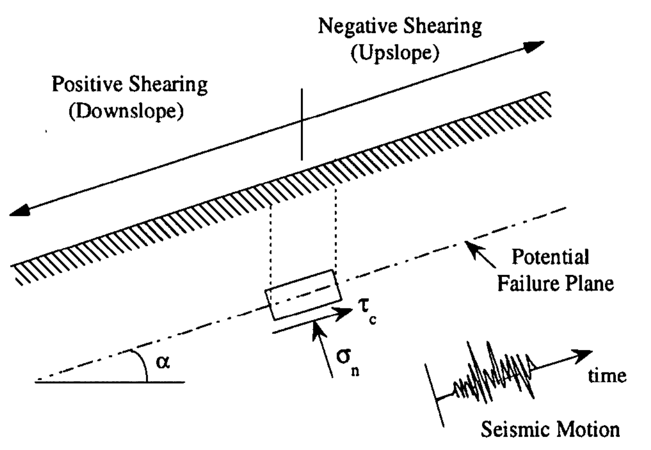
Stability analyses are traditionally performed with pseudo-static methods in which the inertial forces caused by ground acceleration are applied as a horizontal static load following the framework of the limit equilibrium approach. The factor of safety obtained with these procedures hardly satisfies the needs of modern design, based on prescribed levels of performance rather than on a binary safe/fail state criterion. In this context, there is a fundamental need for methods in which ultimate stability and prediction of deformations are addressed simultaneously. In order to achieve this goal it is necessary to model the soil's behavior with a realistic stress-strain-strength relationship that is also capable of handling irregular loading. There are several fundamental aspects that must be considered when evaluating the effectiveness of a constitutive law: the effect of nonlinearity of observed soil behavior, anisotropic stress-strain and strength characteristics resulting from different consolidation stress histories, and the development of excess pore pressure and residual plastic deformation during cyclic loading.

There are numerous effective stress models based on elasto-plasticity theory which include several key elements necessary to model this response, such as hysteretic response and anisotropy (e.g., Prévost 1978, Mróz et al. 1978, Dafalias and Hermann 1982, Iai et al. 1992, Pestana and Whittle 1999). The complexity that allows them to describe soil behavior with high accuracy is also their drawback, because of the high computational cost required for a simple problem such as one-dimensional wave propagation. A second family of models widely used for site response analyses of these slopes follows a more empirical approach by fitting experimental stress-strain response with continuous (Ramberg and Osgood 1943) or piecewise linear (Iwan 1967) expressions for the backbone curve. In most cases, the stress-strain response is decoupled from the generation of excess pore pressures during cyclic loading. More recently, Puzrin et al. (1997) introduced a variant of Iwan's model which included a damage parameter to describe the generation of excess pore pressure during cyclic shearing. Although these models can simulate the anisotropic stress-strain-strength properties associated with a sloping ground surface, invariably it requires the use of different "input model parameters" for each slope inclination. The work presented here uses a new simplified effective stress model for normally and lightly overconsolidated clays, referred to SIMPLE DSS model, which includes the description of small strain nonlinearity, anisotropy resulting from previous consolidation stress history and excess pore pressure generation during cyclic loading.

## SITE RESPONSE ANALYSIS

The results presented here use the computer program AMPLE2000 for one-dimensional site response analysis developed at the Norwegian Geotechnical Institute (Pestana and Nadim 2000). Due to the large areal extent, the response of submarine slides can be analyzed using the infinite slope framework (cf., Fig. 1) as a first approximation. The problem, then, is reduced to the simulation of one-dimensional wave propagation in a layered soil deposit that has been extensively treated in the literature (e.g. Lysmer and Kuhlemeyer 1969, Schnabel et al. 1972, Joyner and Chen 1975, Lee and Finn 1978).

When no seepage forces are present, a soil element within the slope is subject to gravity only and the vertical force can be divided into a component parallel to the slope ($\tau_c$) and another perpendicular to it ($\sigma_n$). For one-dimensional site response analysis, it is customarily assumed that the earthquake shear waves propagate perpendicular to the layers in the direction of the dip and consolidation (i.e., static) shear stress, $\tau_c$. This state of stress can be approximated in the simple shear test apparatus (Fig. 2, Bjerrum & Landva, 1966). This type of testing has been recognized as a useful tool in the study of seismic response of slopes and wave-loading conditions for offshore structures (e.g., Andresen et al., 1979). The constitutive laws needed to describe this type of condition are much simpler than fully six dimensional plasticity theory models.




a) Stress condition in the field

*Fig. 1 Infinite slope under one-dimensional seismic excitation*

The program AMPLE2000 allows dividing the soil profile into any number of layers, each with separate characteristics, including height, material model parameters and preconsolidation pressure. The output includes acceleration, strain and stress time histories at user specified depths, maximum and end-of-shaking profiles of strain, displacement, shear stresses, excess pore pressures and spectral accelerations for 5% damping at prescribed depths in the slope. The layers are modeled as nonlinear shear beams. The finite element formulation requires the solution of the global dynamic equation of motion and the explicit central difference method is used for integration in the time domain.

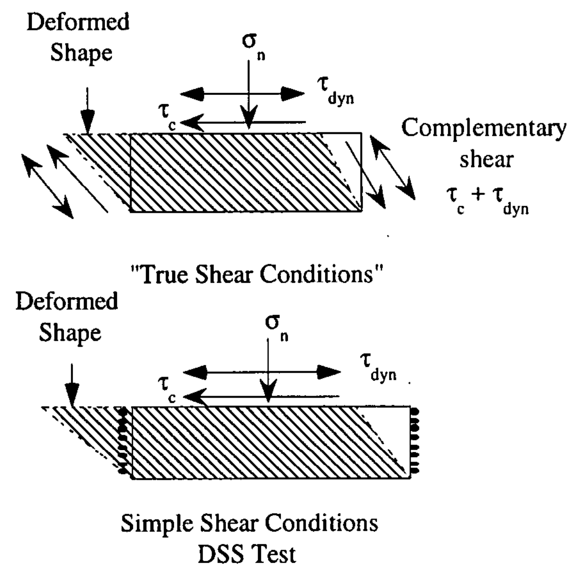



*Fig. 2 Stresses in simple shear conditions.*

<u>SIMPLE DSS Model.</u> SIMPLE DSS is a simplified effective stress model specifically developed for simple shear stress conditions. It is based on an effective stress formulation that allows the simulation of monotonic and cyclic simple shear tests on normally consolidated (NC) to lightly overconsolidated clays. SIMPLE DSS uses the concept of normalized material response in the same way as most effective stress models based on the critical state mechanics framework (e.g., Roscoe and Burland 1968). Monotonic response is described by an anisotropic state surface accounting for the effect of the slope inclination on the observed stress paths. A total of five parameters is needed to simulate monotonic tests and another two describe the behavior during cyclic loading (Pestana et al., 2000). Parameter $\psi$ describes the maximum obliquity for NC specimens (where $\tan\psi$ is the slope of the failure envelope in the normal stress-shear stress space). Parameter $\beta$ controls the amount of pore pressure generated at failure and thus controls the strength at large strains. Both, $\psi$ and $\beta$ can be determined from the effective stress path of a monotonic simple shear test at a 15% to 20% shear strain level. $G_p$ determines the shape of the stress strain curve in first loading and is selected after a short parametric study. Parameter m controls the undrained shear strength at peak conditions and it can be obtained by matching the effective stress path. For cyclic response the constitutive laws incorporate anisotropic hardening to describe different shear strain and stress reversal histories. The use of a

simplified formulation similar to the bounding surface allows a realistic description of the accumulation of plastic strains and the generation of excess pore pressure during cyclic loading. Two additional parameters are required for the simulation of cyclic tests or irregular loading: $\theta$ for the generation of excess pore pressure and $\lambda$ for the accumulation of plastic strains. The parameters are determined independently of each other matching the curves of excess pore pressure and shear strain versus number of cycles in a cyclic stress controlled simple shear test as described by Pestana et al. (2000).

Example slope. In this paper, the proposed framework is illustrated by comparing the response of level ground conditions with that of a slope of 10°. The prototype slope consists of a 20 m deep uniform soft clay deposit with unit weight of 15kN/m$^3$ and an initial shear modulus $G_{max}$ increasing with depth from 50 kPa at the surface to 32400 kPa at 20 m. A single earthquake motion is used for site response analysis. It was recorded during the 1989 Loma Prieta earthquake at Rincon Hill (San Francisco, CA), a site 79.7 km from the fault rupture. The recording has a PGA of 0.092g.

Parameters for the SIMPLE DSS model are given in Table 1 with the value of $G_{max}$ increasing with depth as described above. The values chosen were selected to represent a generic soft clay. Pestana et al. (2000) discuss a procedure to select all parameters for a given clay based on monotonic and cyclic DSS tests.

*Table 1. SIMPLE DSS model parameters for example slope.*

| Parameter | $\beta$ | m | $\psi$ | $G_p$ | $\theta$ | $\lambda$ |
|---|---|---|---|---|---|---|
| Value | 0.35 | 0.50 | 28° | 10 | 25 | 30 |

Figure 3 shows simulations of one dimensional monotonic simple shear tests on a normally consolidated sample for level ground conditions (no consolidation shear stress, $\tau_c = 0$) and a slope of 10° ($\tau_c/\sigma_p$= 0.176, where $\sigma_p$ is the maximum past normal stress) both in a positive and negative direction (by convention downhill and uphill directions respectively). Obviously, for level ground conditions the stress paths are symmetrical and give the same value of normalized undrained shear strength, $s_u/\sigma_p$, of 0.24.

In the case of the slope the presence of an initial downslope shear stress distorts the stress paths, and the direction of the shearing may lead to different behavior. In particular, it is interesting to note that the undrained shear strength changes whether the loading is in the same direction of $\tau_c$ ($s_u/\sigma_p$ = 0.305) or in the opposite one ($s_u/\sigma_p$ = 0.2). The stress-strain curves in Figure 4 also show the difference in the behavior between level and inclined ground conditions. The test with an initial consolidation shear stress and subsequent shearing in the same direction displays a brittle behavior with softening. When the shearing is in the direction opposite to that of the initial $\tau_c$ the behavior is ductile with limited or no softening. The amount of pore pressures developed at peak shear stress conditions also depends on the stress path. When shearing is in the same direction of the initial consolidation shear stress the peak is the highest, but it takes place at the lowest pore pressure and strain levels. On the other hand, if the shearing is in the direction opposite to $\tau_c$ the peak is the lowest, but it is reached at failure conditions when very large pore pressure and strains have developed. This has important implications for the behavior of a slope. Forces acting downward in the dip direction will need to mobilize less strain to reach peak strength and thus reach potentially unstable conditions.

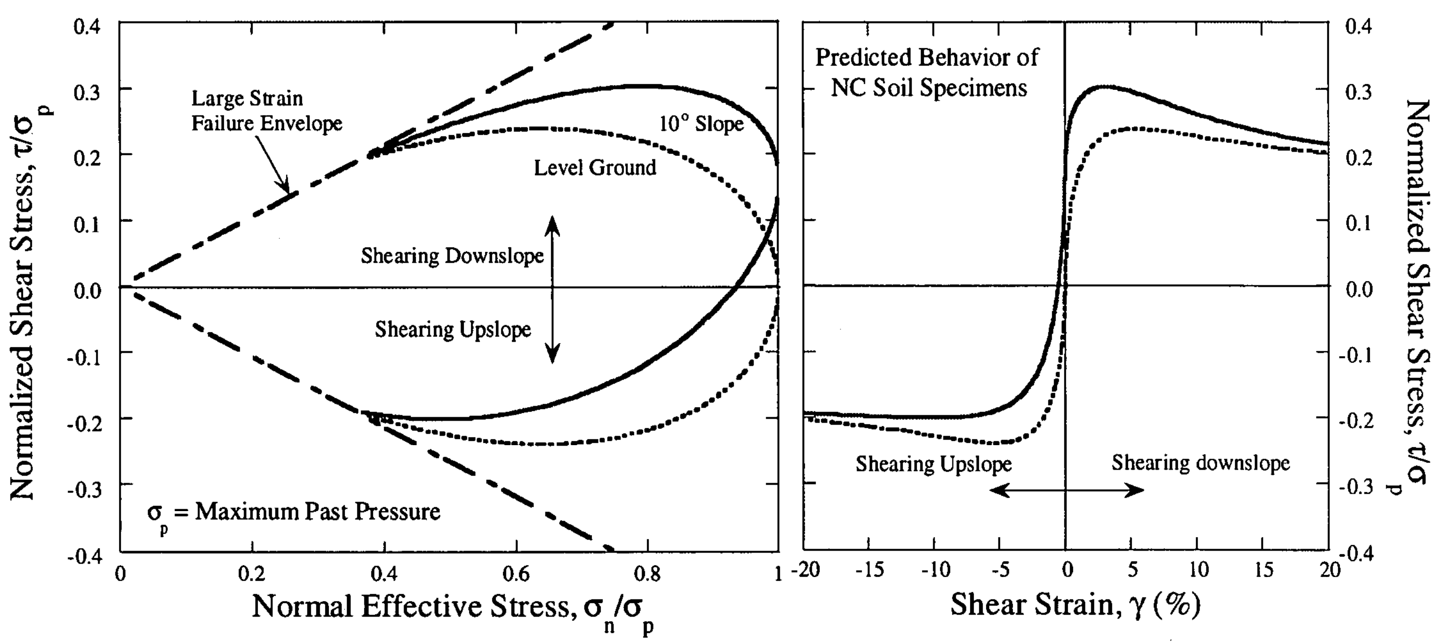



*Fig. 3 Effective Stress paths and stress-strain simulated with SIMPLE DSS for monotonic shearing in the positive and negative directions with and without consolidation shear stress.*

## SLOPE PERFORMANCE

The main issue examined in this paper is the effect of slope inclination on the predicted response as compared to level ground conditions. Results obtained with AMPLE2000 using the SIMPLE DSS model with the same parameters and the same input motion are presented in the following paragraphs. Figure 4 shows the profile of maximum displacement vs. depth for the slope, and for the level ground profile. In the case of the slope the displacement is considerably higher, especially at the top 10-20% of the height.

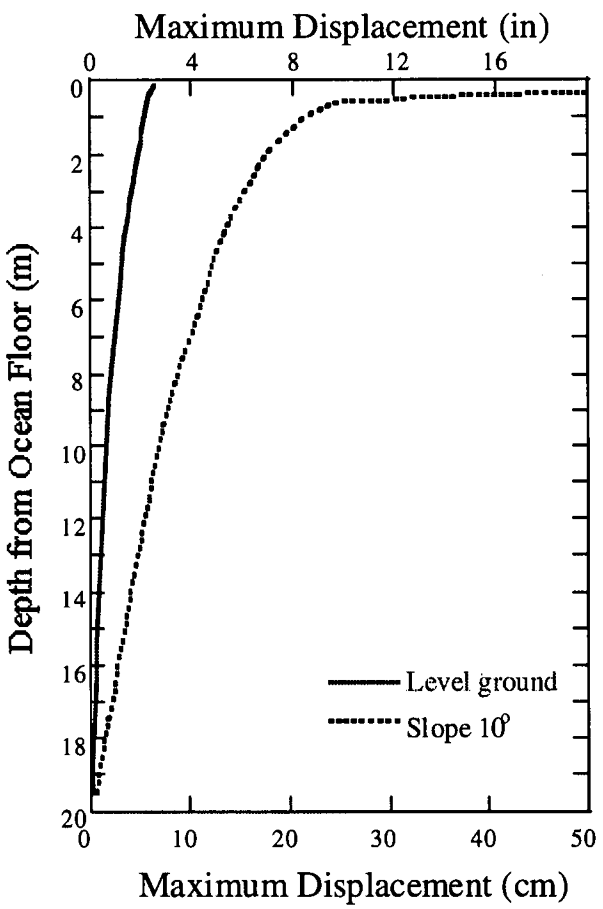



*Fig. 4: Maximum displacement profile along depth.*

When the maximum strain profiles with depth are compared in fig. 5, the same trend can be observed. However, in the case of level ground the end of shaking strains are much smaller than the maximum values. For the sloping ground case, the end of shaking condition is almost indistinguishable from the maximum and was not plotted on the graph for this reason.

When the ground is level and the soil is uniform the direction of loading makes no difference in the response, as for the monotonic tests above. Therefore we expect that the soil itself will experience positive and negative displacements, but that the maximum in one direction will be partially reversed during subsequent shearing. Figure 6a shows the displacement time history at the depth of 1m. There are large spikes in both direction, but eventually the permanent displacement is only a fraction of the maximum. The same takes place at all depths, although not represented in Fig. 6a for reasons of clarity, thus the profiles for maximum and final values are different.

When the ground is sloping, the monotonic tests above demonstrated that the direction of loading largely influences the behavior of the soil. Even if the motion were perfectly symmetrical the response at the surface would not be, because of the anisotropy in the behavior of the soil. In Fig. 6b the displacement time histories for the 10° slope are plotted for various depths. Although the earthquake motion and the soil parameters are the same, the response is very different from that illustrated in Fig. 6a. The most apparent difference is the accumulation of the displacements in the positive –that is downhill –direction with only minor reversals for upslope accelerations. The same trend is observed at all depths and the final values are very close to the maximum values, as observed earlier. For the flat surface condition gravity is acting perpendicular to the earthquake accelerations and its effect is not direct. In the slope part of gravity acceleration is in the same direction of the earthquake accelerations and it actively drives the displacements downward.

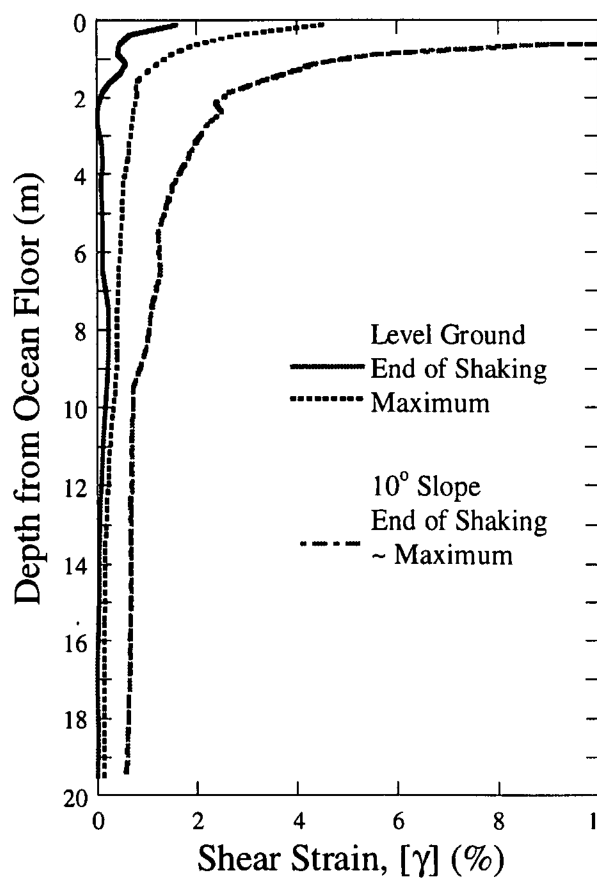



*Fig. 5: Shear strain profile with depth*

Comparing Fig. 6a with 6b it is possible to observe that while in the first case the displacements are mostly negative after 10 s, in the second the displacements are always positive. When the input record is reversed, the level ground response is identical in absolute value. By reversing the input motion and applying it to the 10° slope we obtain the displacement time history in Fig. 7. As can be seen, the maximum response for a slope is vastly dependent on the shaking direction (sometimes referred to as above horizon vs. below horizon analyses). In this particular case, the peak ground acceleration is acting in the downslope direction and the lateral displacement increases by 40%.

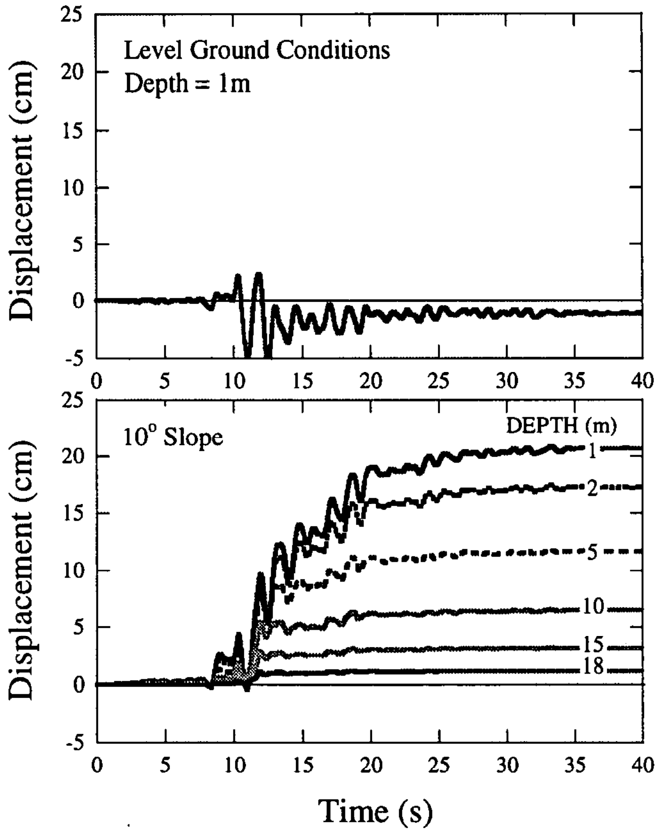



*Fig. 6: Lateral displacement time histories (a) for level ground conditions at the depth of 1m; (b) for a slope of 10° at multiple depths.*

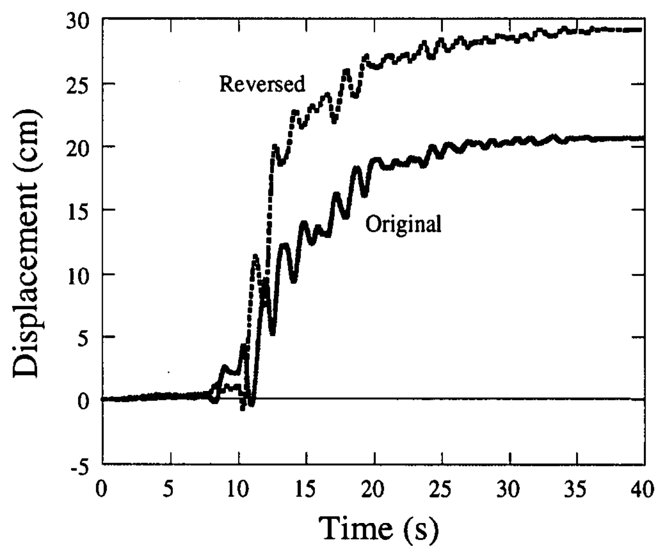



*Fig. 7: Comparison of lateral displacement time histories for the earthquake record applied in the original and reversed direction (above vs. below horizon analysis).*

Figure 8 shows the profile of excess pore pressure ratio ($\Delta u_{excess}/\sigma_{vo}$) for both cases along the depth. There are only minor differences except at the top, where failure conditions are reached in the slope. For the parameters assigned to this soil failure is reached with a pore pressure ratio of 0.63.

The acceleration response spectra at the depth of 1m for level ground and the slope are compared with the bedrock in Fig. 9. For the level ground conditions spectral accelerations at periods larger than 0.7s are substantially amplified with a peak close to the nonlinear site period (T = 1.3 s). The predominant period for the soil profile based on initial (i.e., $G_{max}$) conditions is less than 1 second. For the slope case, it is possible to observe a significant amplification with respect to rock motion in the range of 0.7- 4 seconds. It is important to note that the amplification is smaller than that of the level ground due to significant softening and larger strains in the top soil. In addition, the period at peak amplification is slightly increased to approximately 1.5 seconds as compared to 1.3 seconds in the case of level ground conditions.

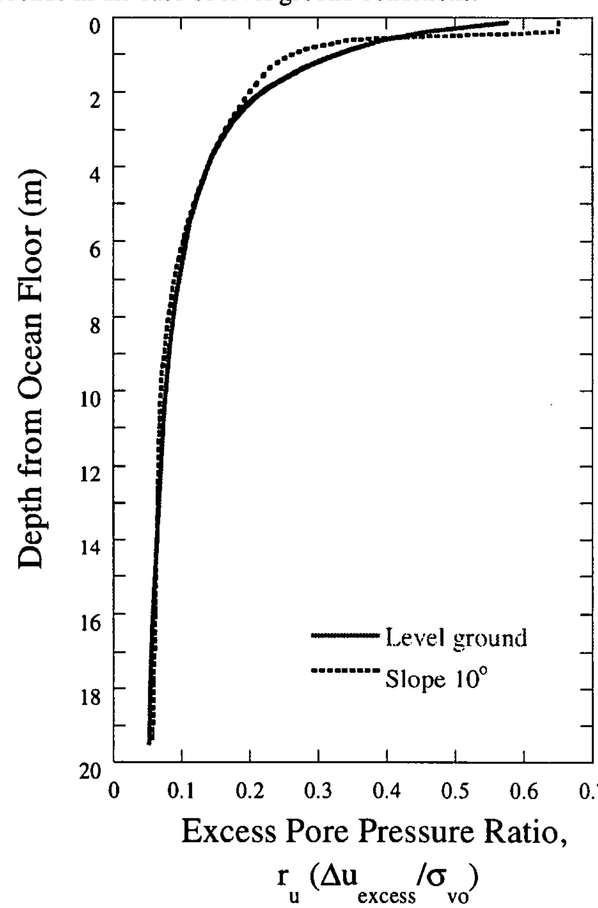



*Fig. 8 Profile of pore pressure ratio with depth.*

CONCLUSIONS

Observed behavior in simple shear tests has shown that the presence of an initial shear stress (equivalent to sloping ground conditions) can significantly influence the response of the soil in monotonic shearing. The same also holds true for the irregular loading of earthquakes. Comparison of the results obtained with AMPLE2000 for the simple example in this paper illustrates the importance of incorporating the consolidation shear stresses (and thus shear stress history induced anisotropy) into account. In the case of the slope the displacement is accumulating downhill with only minor reversals for uphill accelerations. At the end of shaking much larger strains are accumulated in the slope than in the flat surface case. Since earthquake accelerations are not symmetrical the direction in which the recorded motion is applied is also important, because it can lead to different responses.

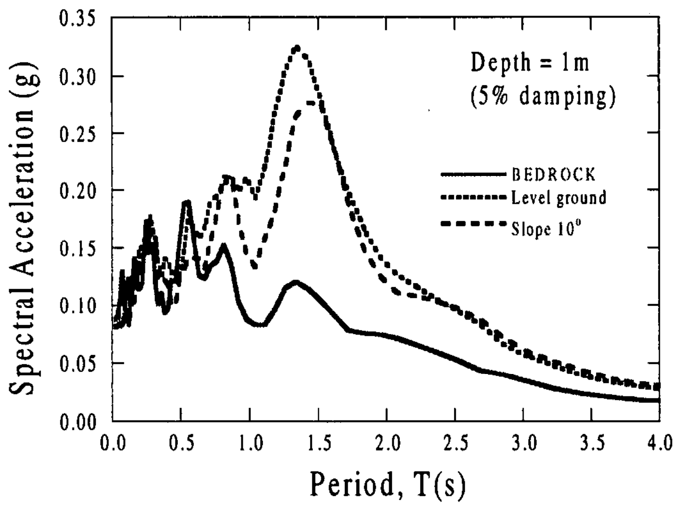



*Fig. 9: Comparison of acceleration response spectra at the bedrock for level ground and sloping conditions.*

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